CHAPTER 4

HOW IS GROUND IMPROVEMENT DESIGNED?

Design Considerations and Parameters

After it is determined that ground improvement is required, a treatment method must be selected and an improvement program designed. The project design and performance requirements will dictate some of the design parameters, including the required stability and the allowable deformation of treated ground under static and dynamic loading. The subsurface conditions will set other design criteria, such as the suitability of different ground improvement methods and the required depth and areal extent of treatment. Collectively, these factors will determine the level of improvement required to assure satisfactory performance. Site constraints will also play a role in design, as will the construction schedule and the construction budget. Finally, the availability of experienced or specialty contractors in the area will be a design consideration.

Design and Performance Requirements. Different structures will have different performance requirements; for example, a linear structure like a bridge may have different displacement limitations than a settlement-sensitive isolated building. In determining the level of improvement required, the following questions should be considered:

1. Is the improvement for an existing facility or a proposed facility?
2. How much settlement is the structure able to tolerate under normal service conditions? How much movement or settlement is tolerable during a natural hazard such as an earthquake or a flood?
3. Is the facility a critical or a non-critical structure? A critical structure could be a navigation lock where closure of the facility could result in serious economic losses or a dam where failure could cause significant loss of life or property. A non-critical facility could be a warehouse, where significant damage would be inconvenient, but not critical or life-threatening.

4. Can the facility tolerate the anticipated seepage or would it cause economic losses or danger of erosion and piping?
5. How much resistance to liquefaction is needed? Should a "two-level" mitigation strategy be used whereby sufficient remediation is proposed to: (1) avoid significant damage and loss of serviceability under the design earthquake and (2) avoid catastrophic failure, while allowing repairable damage, in the maximum credible earthquake (Mitchell et al., 1998)?

Site constraints. Site constraint considerations can be addressed by the following questions:

1. How large is the area that needs to be treated?
2. Is the site large or small? Is it open or constrained by structures or utilities?
3. Are there nearby buildings that are sensitive to vibrations?
4. Will property easements from adjacent sites be necessary to complete the ground improvement, e.g. for soil nailing or micro-piles?

Subsurface conditions. Answers to the following questions will aid in selecting suitable methods and determining the size and depth of the treatment zone:

1. What type of soil needs to be improved? What methods are appropriate for improving it?
2. At what depth and how thick is the layer that needs to be treated? How far outside the footprint of the structure does the layer need to be treated?
3. Is the layer saturated? At what depth is the ground water table?
4. Is there more than one layer that needs to be treated, such as a loose fill overlying a soft clay layer? Is a different method needed for each layer that needs to be treated, or can one method treat all the layers that need to be improved?

Scheduling. Construction scheduling can restrict the potentially applicable ground improvement methods. Certain methods produce immediate improvement (e.g. vibroflotation), while others require time (e.g. wick drains). Other methods produce an initial improvement and then a continuing strength gain with time (e.g. explosive compaction, methods involving cementation reactions). The improvement method selected must be compatible with the time available for improvement.

Budget and availability of contractor. The selection of a ground improvement method will also depend on the construction budget and the funds available for improvement. If plenty of free fill is available, use of a buttress may be a cost effective improvement technique. At premium urban sites, the cost of more expensive improvement methods may be relatively small when compared to real estate costs. If a specialty contractor is located near the site, selection of a proprietary ground improvement method may be cost effective because of a relatively small mobilization charge.

Design Procedures

With the aid of answers to the foregoing questions, the following steps can be followed to design the ground improvement program:

1. Select potential improvement methods.
2. Develop and evaluate remedial design concepts.
3. Choose methods for further evaluation.
4. Perform final design for one or more of the preliminary designs.
5. Compare final designs and select the best one.
6. Field test for verification of effectiveness and development of construction procedures.
7. Develop specifications and QA/QC programs.

These steps are discussed in more detail below.

Select potential improvement methods. A preliminary screening and evaluation of methods can be made using Tables 2, 3, and 4 in Chapter 3. A list of potentially applicable methods for a particular ground improvement purpose can be developed using Table 2. The list can be refined by using Tables 3 and 4 to select methods that should be suitable in light of the particular site constraints.

Develop and evaluate remedial design concepts. Preliminary designs can be developed for each improvement method selected in the previous step. Tentative layouts and treatment points for each method can be developed using Tables 3 and 4, and/or from propriety or empirical guidelines and design programs offered by specialty contractors. The tentative size and location of the treatment zone can be established using empirical guidelines, which are discussed below in "Design Recommendations." If the design includes retrofitting a structure, the improvements to existing foundation elements should be determined, and/or new foundation elements should be designed.

Analyses should be performed for each preliminary design to determine if the treated zone will be improved sufficiently to meet the design and performance requirements. For non-critical structures, the analyses may be as simple as confirming that the factors of safety are adequate when computed using the anticipated properties for the improved soil. However, detailed ground deformation and foundation loading analyses may be required for critical or complex structures. These analyses require information on the geometry and properties of the treatment zone for each improvement method. Preliminary cost estimates can also be developed using Table 5 to aid in selecting methods for further evaluation.

Choose methods for further evaluation. The preliminary designs can be compared to determine which methods appear to be the best alternatives for the particular site. Further analysis can be done for each of these options.

Develop tentative final designs for the selected preliminary designs. Detailed design and cost estimates are developed for one or more of the selected preliminary designs. The location, size, shape and required properties of treatment zones or foundation improvements are determined. This stage includes determining locations and depths of treatment and developing construction details for the foundation improvements. Methods for evaluating the post-treatment results in the field are developed. Analyses are performed for the final designs to confirm that the anticipated performance of the facility will be satisfactory.

Compare final designs and select the best one. The final design plans and cost estimates are analyzed to determine the best scheme for improving the site or facility. The final selection is based both on cost and on the expected performance of the facility after improvement, constructability, the time available for construction, and the availability of contractors to perform the work.

Field testing for design verification and development of construction procedures. For most projects, a field testing program should be developed and executed to verify that the required improvement can be obtained using the proposed method. The design can be adjusted during this phase to optimize the spacing of the treatment locations so the required improvement can be obtained in an efficient manner.

Develop specifications and QA/QC programs. Construction specifications and QA/QC programs will be required for the design that will be implemented. The specifications can be either procedural or end result, however, the QA/QC program should be consistent with the type of construction specifications. These issues are discussed in more detail in the following chapter.

Design Issues

There are certain design problems that are specific to certain ground improvement methods, while others are general and apply to most methods. In general, ground improvement designs are based on empirical guidelines rather than rigorous design procedures. Some methods are proprietary and can only be designed and implemented by specialty contractors. Most require extensive field testing programs before the design can be finalized. Some are still being developed, so it may sometimes be difficult to write unambiguous and enforceable specifications and QA/QC programs.

Some of the design problems specific to different methods or applications are summarized below.

Prefabricated Vertical (PV) Drains (Wick Drains): According to ASCE (1997), PV drains have performed well in many past projects mainly because they are designed conservatively. When PV drains are designed to function near their maximum capacity, the installations will need to be monitored carefully. The drain capacity could be the limiting factor in cases where PV drains are designed for sites where there are deep compressible layers with surcharge loading. Before using PV drains below a depth of 45 m, a specialist should be consulted. PV drains have been used for mitigation of liquefaction risk in a few cases; however, little research has been performed to quantify the extent of improvement that can be obtained in this application.

Soil Nailing: There have been inconsistencies in the design methods for soil nailed walls (Xanthakos et al., 1994). It is recommended that the Manual for Design and Construction Monitoring of Soil Nail Walls (FHWA, 1996b) be used, as it synthesizes current design and construction methods into a comprehensive and consistent guideline procedure. Worked design examples are included in the manual. A companion manual for construction monitoring is also available (FHWA, 1996c).

Micro-piles: When conventional piles are closely spaced, the nominal capacity of each pile is reduced to account for a group effect. In contrast, closely spaced pin piles have been reported to have higher capacity than widely spaced piles, particularly when the piles are reticulated, i.e. intertwined (Xanthakos et al., 1994). This positive group effect is not routinely exploited in design. However, there is also no reduction to account for a group effect as is done in conventional pile design.

Stone columns/Gravel drains: When gravel drains are used for dissipation of excess pore pressure, it is difficult to predict the permeability that can be obtained. During installation, there is mixing between the stone and the in-situ soil, so the final drain contains a mixture of soil and stone. Different studies have estimated that the in-situ soil comprises about 20% of the completed stone column (Boulanger et al., 1998). It is also difficult to measure the permeability properties of stone columns in the field.

Seismic applications: When designing ground improvement to reduce the risk of liquefaction or lateral spreading, the primary concern is limiting the deformations of a supported structure to acceptable levels. In order to limit deformations, it is first necessary to have adequate ground strength to resist overall failure of the ground and structure.

There are numerous factors which influence the stability and deformation of improved ground zones and structures during and after an earthquake, as described by Mitchell et al. (1998). The size, location and type of treated zone influences the behavior of the improved ground and the supported structure. Migration of pore pressure from an untreated zone into an improved zone can reduce the strength in the improved zone. Improved ground may amplify the earthquake motion, resulting in more severe loading on a supported structure. The maximum inertial forces that act on the improved ground and the structure may act at different times, causing a complex soil-structure interaction problem. In cases where improved ground is located in sloping areas, there may be additional forces imposed on the improved ground zone if the surrounding unimproved ground undergoes lateral spreading. Some of these factors can be incorporated into complex analytical models, but most of them have not been incorporated into simplified methods of analyses.

Design Recommendations

Depth of treatment: For liquefaction mitigation, the depth of treatment generally should extend to the bottom of the layer that requires improvement, particularly for large or heavily-loaded structures. For lightly-loaded structures, it may not be necessary to treat the entire liquefiable layer, however, design procedures for an improved "crust" over liquefiable soils are not well established. For free-field conditions or lightly-loaded structures, Ishihara (1985) presents correlations between the minimum thickness of a non-liquefiable surface layer, the maximum thickness of an underlying liquefiable layer and surface manifestations of liquefaction. For several sites in Japan subjected to maximum accelerations of about 0.2g, liquefaction damage was observed when the crust thickness was less than 3 m. For sites where the crust thickness was less than 3 m, more damage was observed if the liquefiable layer was greater than 3 m in thickness. Youd and Garris (1995) performed a similar study on additional sites and concluded that Isihara's 1985 criteria were valid for sites that are not susceptible to lateral spreading or ground oscillation. Naesgaard et al. (1998) developed a simplified procedure for determining the response of a foundation placed on an existing cohesive crust if the underlying layer liquifies. This method was mentioned in Chapter 2.

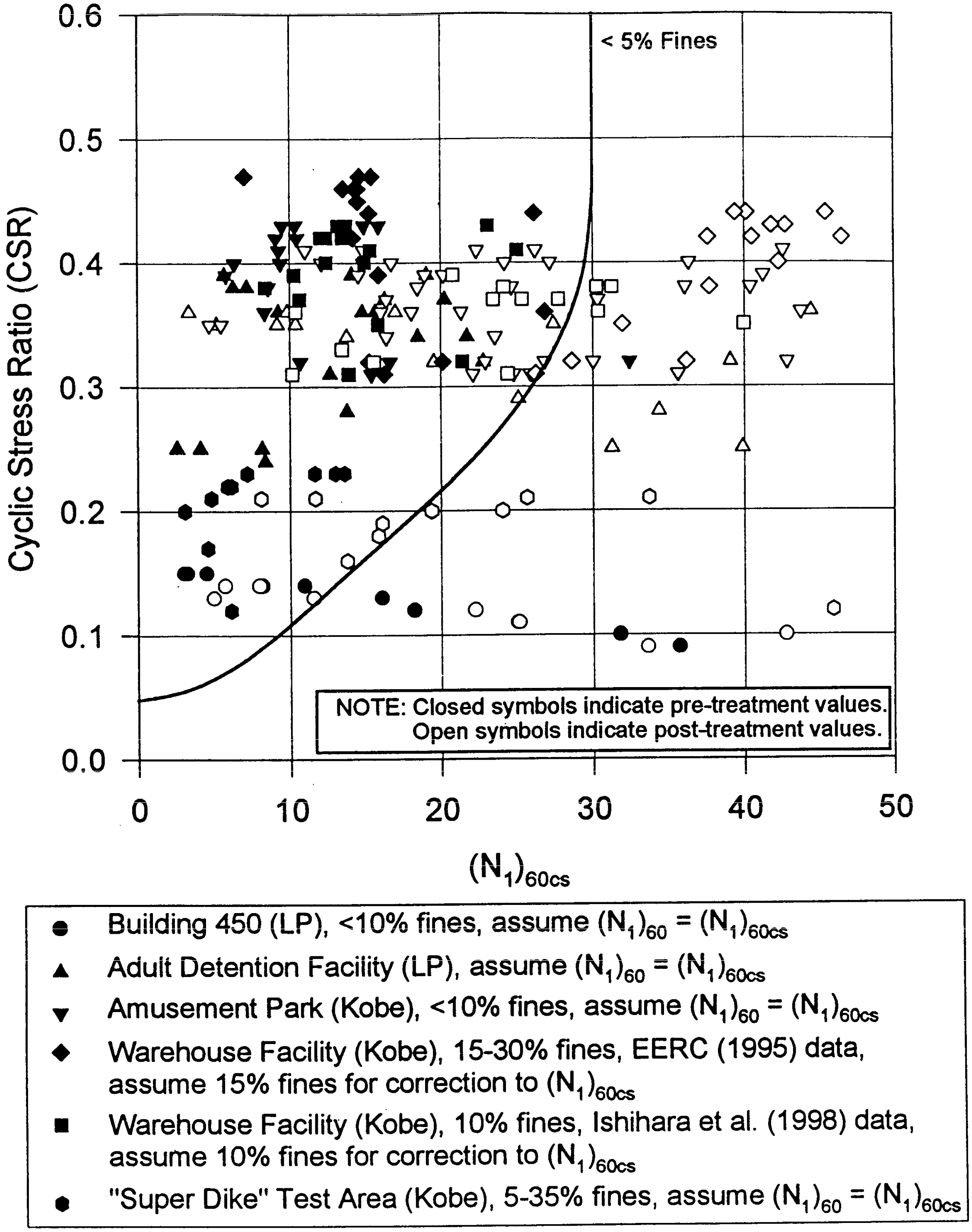
For "conventional" ground improvement applications, the depth of treatment should extend either to the depth of influence of the structure or to the bottom of the layer requiring improvement. The approximate 2:1 load spread method can be used for a first estimate of the depth of influence of the structure. The load spread method assumes that the stress from a foundation spreads out beneath the structure on lines with a slope of 2 vertical to 1 horizontal. The average stress increase at a depth z, assuming rectangular foundation dimensions L and B and an average pressure of q, can be calculated by the following equation:

$$\Delta\sigma_z = \frac{qLB}{(L+z)(B+z)}$$

If more accuracy is needed, a Boussinesq or Westergaard analysis can be used.

Areal extent of treatment: For liquefaction protection, the treatment zone should generally extend outside the perimeter of the structure at least a distance equal to the thickness of the treated layer. The performance of sites where space constraints prevented implementation of this recommendation are discussed in Chapter 6. For "conventional" applications, the treatment zone should extend outside the perimeter at least a distance equal to half the thickness of the treated layer. This guideline accounts for the stress increase beneath a foundation based on the approximate 2:1 load spread method.

Seismic remediation: Liquefaction potential assessment curves (Seed et al., 1984, NCEER, 1997) appear useful for design of ground improvement by densification in seismic areas. The effects of ground improvement on liquefaction potential for five improved sites that were shaken in the 1989 Loma Prieta or the 1995 Hyogo-ken Nambu (Kobe) earthquakes are shown in Figure 44. The liquefaction-no liquefaction boundary curve shown is the consensus curve adopted in NCEER (1997) for clean sand and a magnitude 7.5 earthquake. All data points have been corrected for fines content, overburden pressure and earthquake magnitude according to the NCEER (1997) recommendations to give the equivalent $(N_1)_{60cs}$ and cyclic stress ratio (CSR) values shown. The closed and open symbols on the figure indicate pre- and post-treatment SPT $(N_1)_{60cs}$ values, respectively. The percentage of fines, if known, is shown on Figure 44 for each facility. If the percentage of fines was not known, the $(N_1)_{60}$ value was assumed to equal $(N_1)_{60cs}$. For the most part, the liquefiable layers were improved from the "liquefaction" (left) to the "no liquefaction" (right) side of the liquefaction potential curve. With the exception of the Kobe Port Island Warehouse, little or no deformation was reported at the sites after shaking. From these data, it appears that liquefaction effects will be minor if the supporting ground is improved by densification to the "no liquefaction" side of liquefaction potential curves for CSR values less than about 0.3, and ground deformations will be reduced significantly for higher levels of shaking. For design using the liquefaction potential curve, the CSR and the percentage of fines, the minimum required $(N_1)_{60cs}$ can be determined throughout the potentially liquefiable layer.



Notes: 1. The CSR values were adjusted to equivalent CSR values for the M=7.5 base curve using the magnitude scaling factor proposed by Idriss (1997).

2. $(N_1)_{60}$ values for were corrected to clean sand $(N_1)_{60cs}$ values based on NCEER (1997).

Figure 44. Effect of ground improvement on liquefaction potential for sites that were shaken in the 1989 Loma Prieta and 1995 Hyogo-ken Nambu (Kobe) earthquakes.